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Assessment of Liquefaction Potential and Post-Liquefaction Behavior of Earth Structures: Developments 1981-1991

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Assessment of Liquefaction Potential and Post-Liquefaction Behavior of Earth Structures: Developments 1981-1991

(State of the Art Paper)

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SYNOPSIS Significant developments in procedures for evaluating liquefaction potential and the post-liquefaction behaviour of earth structures are reviewed for the period 1981-1991. Particular attention is paid to the re-evaluation of the liquefaction induced slide in the San Fernando Dam following the 1971 earthquake. The findings of this study have had a major impact on engineering practice.

INTRODUCTION

Developments in the evaluation of liquefaction potential and of the post-liquefaction behaviour of earth structures since the first conference was held in 1981 will be reviewed. It was a period of major developments which have had significant impact on engineering practice. Perhaps the most significant event was the re-evaluation of the liquefaction induced slide in the San Fernando Dam which occurred following the 1971 earthquake. This study was conducted under the sponsorship of the U.S. Army Corps of Engineers, Waterways Experiment Station in Vicksburg, Mississippi. Various aspects of the study have been reported by Castro et al. (1989), Seed et al. (1988) and Vasquez-Herrera and Dobry (1989). The most important outcome of the study was the development of two procedures for measuring or estimating the steady state strength of liquefiable soils and an appreciation of the great difficulties surrounding such an enterprise.

The Becker penetration test was introduced by Harder and Seed (1986) for assessing the liquefaction potential of gravels. Stokoe et al. (1989) demonstrated the applicability of surface waves for assessing the liquefaction potential of sands, a technique equally applicable to gravels.

The development of the seismic cone has allowed the measurement of shear wave velocity during cone penetration tests and has greatly facilitated the use of shear wave velocity as an index of liquefaction resistance. Liquefaction assessment charts based on normalized shear wave velocity were proposed by Robertson (1990) and extended by Finn et al. (1990a).

The standard penetration test (SPT) was standardized to an energy level of 60% of the free fall energy of the hammer and the liquefaction assessment chart based on $(N_1)_{60}$ is now the standard used in engineering practice (Seed et al., 1985). The cone penetration test (CPT) has emerged as an alternative to the SPT. The impact of fines on the penetration resistance and liquefaction potential is still a controversial matter which is being slowly clarified.

The use of the Chinese criteria by Wang (1979) in practice for assessing the liquefaction potential of weak plastic soils has shown that the extent of liquefaction can be very sensitive to variations in the parameters used in these criteria and procedures have been developed to take the uncertainties into account in a reasonable way.

Finally, a finite element procedure has been developed for assessing the consequences of liquefaction by predicting the deformed shape of the earth structure after liquefaction. This procedure is incorporated in the computer program TARA-3FL (Finn and Yogendrakumar, 1989). The ability to predict the deformations after liquefaction allows the designer to see the consequences of liquefaction realistically and to plan selective and efficient remediation procedures. It is estimated that use of this type of analysis on two embankment dams requiring remediation has resulted in major savings.

All these developments are reviewed in detail below.

LIQUEFACTION POTENTIAL BY STANDARD PENETRATION TEST (SPT)

Seed's liquefaction assessment chart (Fig. 1) is the primary tool in current practice for determining whether potentially liquefiable soils will liquefy or not (Seed et al., 1985). The soil conditions are defined by the normalized standard penetration resistance $(N_1)_{60}$ and the effects of the earthquake by the average cyclic stress ratio τ_{av}/σ'_o where τ_{av} is the average or effective shear stress mobilized by the earthquake shaking and σ'_o is the vertical effective confining pressure. Procedures for determining $(N_1)_{60}$ and τ_{av}/σ'_o has been presented by Seed et al. (1985). The chart in Fig. 1 applies to earthquakes of magnitude M = 7.5 and charts for other earthquake magnitudes can be generated by using appropriate scaling factors (Seed et al., 1985). A set of curves for clean sands for different earthquake magnitudes are shown in Fig. 2.

There are two important limitations associated with Fig. 1. The field data correspond to level ground conditions with no initial static shear stresses on horizontal planes and to effective overburden pressures less than 150 kPa. Seed (1983) outlined procedures for making corrections when these conditions are violated.

The first estimate of the liquefaction resistance of a soil element in a dam or slope is determined using

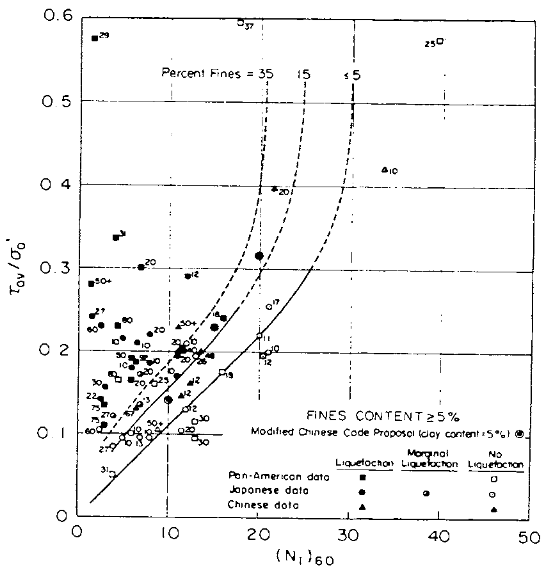

Fig. 1. Seed's Liquefaction Assessment Chart (1985)

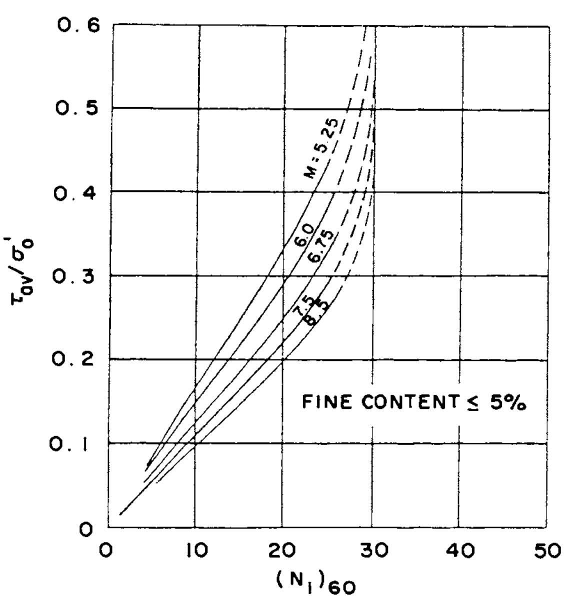

Fig. 2. Liquefaction Assessment Chart for Various Earthquake Magnitudes (Koester and Franklin, 1985)

the in situ $(N_1)_{60}$ penetration resistance and the appropriate curve for critical conditions in Seed's liquefaction assessment chart. This resistance must then be corrected for deviations from the standard conditions of the database underlying the chart. A typical element in a slope, for example, will carry a static shear stress τ_{st} on the horizontal plane and therefore has an initial shear stress ratio $\tau_{st}/\sigma'_o = \alpha$. A correction factor K_α is established for various values of α by laboratory tests. As an illustration, the K_α factor for Folsom gravels is shown in Fig. 3

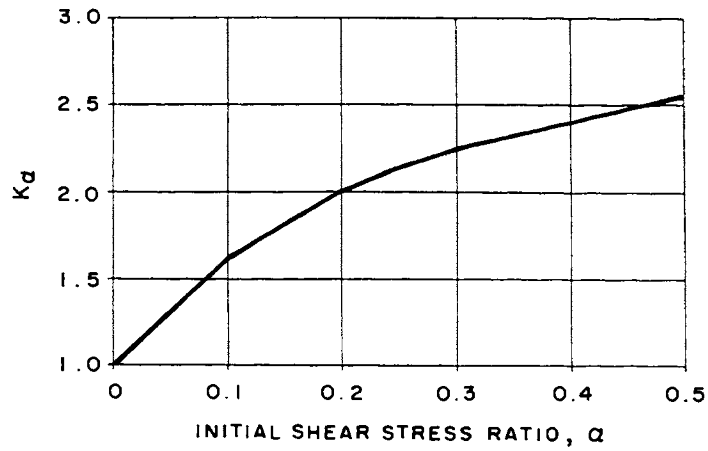

Fig. 3. Folsom Gravel K_α Factors (Hynes et al., 1988)

(Hynes et al., 1988). Note that, as is commonly assumed in practice, initial static shear increases liquefaction resistance substantially. However, this increase applies to resistance to cyclic mobility rather than to liquefaction, that is to non-contractive materials.

Vaid and Chern (1985) have shown for contractive materials that initial static shear decreases the resistance to cyclic loading and increases the contractiveness of the sand and the strength drop after peak in undrained shear. This suggests that for contractive soils, the cyclic strength should be reduced not increased for the effects of static shear. K_α factors less than unity have been reported by Marcuson et al. (1990) and Seed and Harder (1990). Typical value of K_α may be found in Fig. 4 for a range in relative densities.

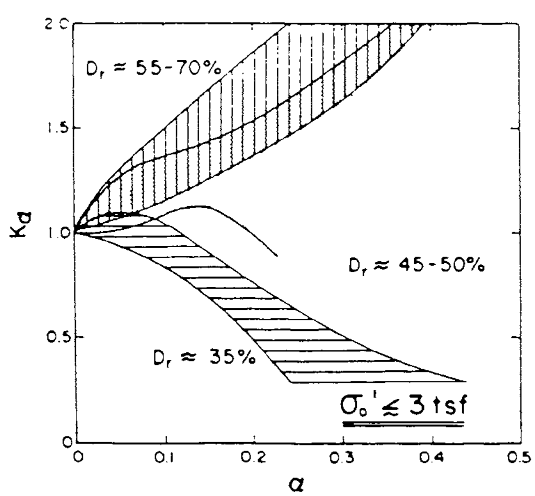

Fig. 4. Ranges in K_α for Various Relative Densities (Seed and Harder, 1990)

For effective overburden pressures other than in the range 100-150 kPa a correction factor K_σ is used.

Values of K_σ for Folsom gravels are shown in Fig. 5. Increasing confining pressure can lead to a substantial reduction in resistance to cyclic loading.

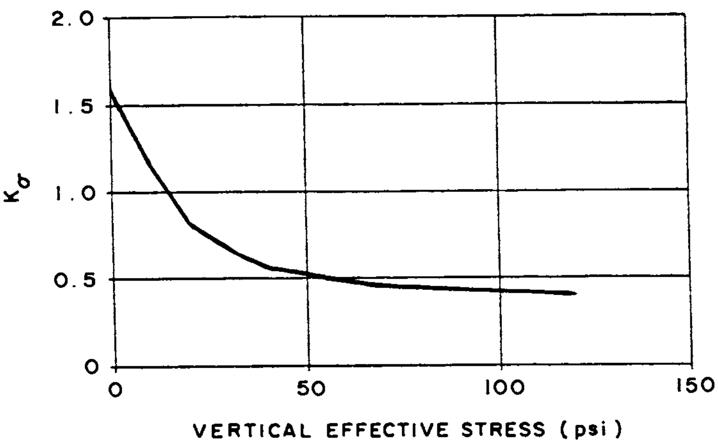

Fig. 5. Folsom Gravels K_σ Factors (Hynes et al., 1988)

The field data in Fig. 1 shows a strong dependency of liquefaction resistance on fines content. But, surprisingly, there is considerable confusion about the effect of fines on liquefaction resistance. Part of the problem depends on the manner on which the comparison is made. In the case of the field data, for the same normalized penetration resistance $(N_1)_{60}$ the liquefaction resistance increases with increasing fines content. What the soils being compared have in common is the $(N_1)_{60}$ value. If the basis for comparison is changed the conclusion may well be different. For example, Troncoso (1990) compared the cyclic strength of tailings sands with different silt contents ranging from 0 to 30% at a constant void ratio. On this basis of comparison he found that cyclic strength decreased with increased silt content (Fig. 6). Kuerbis and Vaid

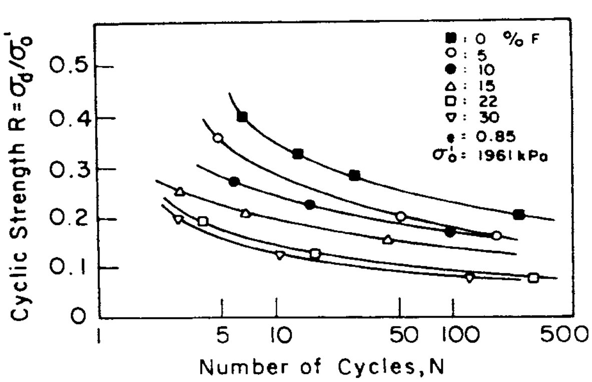

Fig. 6. Variation in Cyclic Strength with Fines Content (Troncoso, 1990)

(1989) studied the effects of fines using samples deposited in a manner that replicated field deposition conditions. They found for the sand tested that up to 20% fines could be accommodated within the sand skeleton and that samples with the same sand skeleton void ratio had the same cyclic strength for fines content less than 20%.

Another way of looking at the field data is to note that the penetration resistance of silty sands is somewhat lower than for clean sands at the same cyclic strength. Corrections based on the curves in Fig. 1, can be applied to the $(N_1)_{60}$ values measured in silty sands and then the curve for clean sands may be used to determine cyclic strength for the corrected $(N_1)_{60}$ value. Such corrections have been widely used.

Seed et al. (1988) introduced corrections on a different basis. He proposed that, for equal relative densities, values of $(N_1)_{60}$ determined for silty sands could be corrected to equivalent clean sand values by adding increments given in Table I.

TABLE I. Corrections to Measured $(N_1)_{60}$ for Fines Content

Fines Content	$(\Delta N_1)_{60}$
10%	1
25%	2
50%	4
75%	5

These increases in $(N_1)_{60}$ imply that for equal relative density the cyclic strength increases with an increase in fines content. In the analysis of the slide deformation in the Lower San Fernando Dam (Seed et al., 1988) a correction of 2 was made to the average measured $(N_1)_{60}$ value of 12.5 for the effect of the estimated fines content of 25%. It is important to note that the correction for constant relative density is not in conflict with data in Fig. 1. The basis of comparison is quite different in both cases. The $(\Delta N_1)_{60}$ that would be deduced from Fig. 1 would be based on equal cyclic strength whereas that in Table I is based on equal relative density.

The different methods for considering the effects of fines are summarized in Table II.

TABLE II. Effects of Fines on Cyclic Strength

	Criterion of Equivalency Clean and Silty Sand	Effect of Fines on Cyclic Strength
1.	Same $(N_1)_{60}$	Increase
2.	Same void ratio	Decrease
3.	Same void ratio in sand skeleton	No change while fines content can be accommodated in sand voids
4.	Same relative density	Increase

At the present stage of development of research on the effects of fines the use of criterion 1 and Seed's chart in Fig. 1 seems the most reliable way to estimate the effects of fines on cyclic strength for engineering practice.

LIQUEFACTION ASSESSMENT BY CONE PENETRATION TEST (CPT)

Electric cone tip penetration resistance (Q_c) has been measured at few sites as yet where the occurrence or non-occurrence of liquefaction during earthquakes have been documented. Therefore the data base on cone penetration resistances at liquefied sites has been extended by converting the well documented correlation between liquefaction potential and standard penetration resistance to cone penetration resistance.

There have been a number of studies on the correlation between cone resistance and standard penetration resistance (Douglas et al., 1981; Robertson et al. 1983; Seed and De Alba, 1986). Figure 7 indicates that the ratio of cone resistance to blow count increases with increasing mean grain diameter (Robertson et al., 1983).

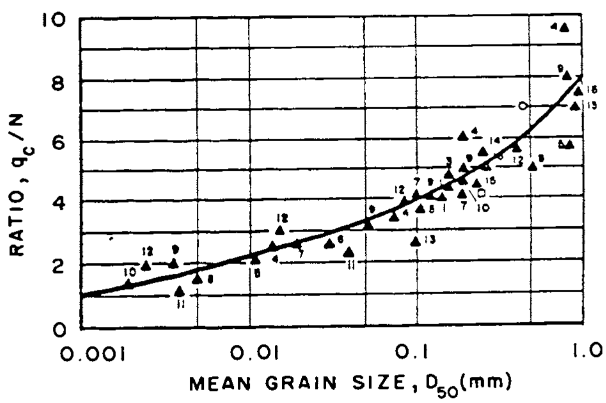

Fig. 7. Relationship Between q_c/N_{SPT} and Medium Grain Size D_{50} (Robertson et al., 1983)

The liquefaction assessment chart in Fig. 1 based on $(N_1)_{60}$ can now be converted to a chart based on Q_{c1} as shown in Fig. 8.

Corrections to the normalized cone penetration resistance with fines $(Q_{c1})_f$ can be established from Fig. 8 that may be used to convert penetration resistance data in silty clays to equivalent penetration resistances of clean sand with similar liquefaction potential. The correction is given by

$$Q_{c1} = (Q_{c1})_f + \Delta Q_{c1} \qquad (1)$$

where ΔQ_{c1} are given in Fig. 9.

Robertson and Campanella (1985) and Mitchell and Tseng (1990) have presented reviews of the assessment of liquefaction potential using the CPT.

It is possible to estimate crudely the fines content directly from cone penetration test data. Fig. 10 presents the latest soil classification chart

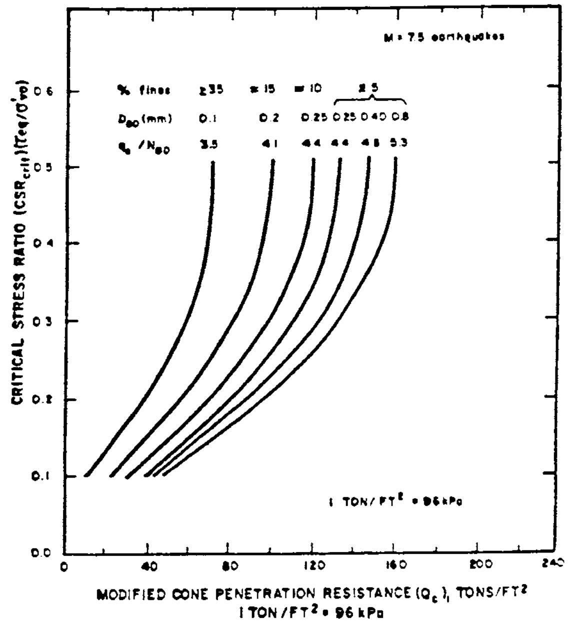

Fig. 8. Liquefaction Assessment Chart Based on Normalized CPT (Seed and De Alba, 1986)

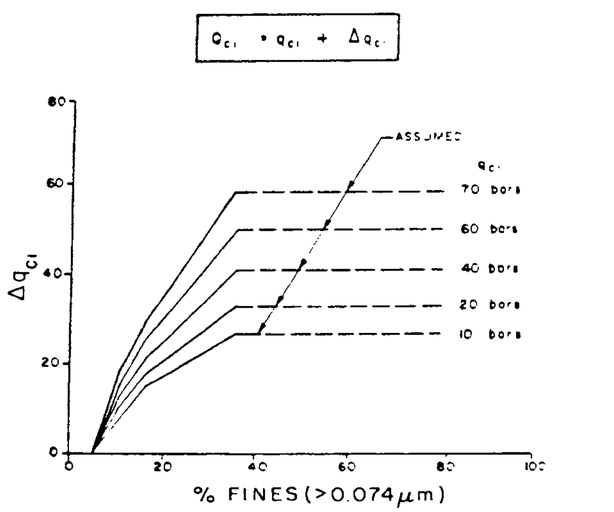

Fig. 9. Correction for Fine Content to CPT Data (Finn et al., 1990a)

based on normalized CPT data. Soils that fall in zone 6 are generally clean sands or silty sands with a small amount of fines. Soils that fall in zone 5 are silty sands and sandy silts that generally have fines contents greater than about 15%. The classification can be improved by using the piezocone which allows the measurement of the porewater pressure ratio (Robertson, 1990).

Finn et al., (1989) attempted to correlate fines content and the time for 50% dissipation (T_{50}) of the pore pressures developed during penetration using data from piezocone penetration tests in the Fraser Delta in British Columbia (Woeller, 1988). The results in Fig. 11 suggest that for T_{50} > 50 sec the fines content is

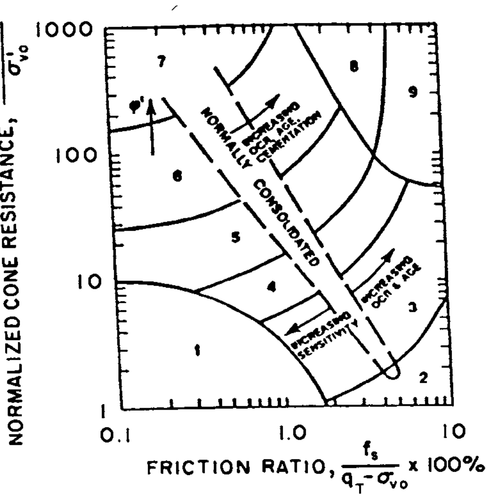

Fig. 10. Soil Classification Chart Based on Normalized CPT Data (Robertson, 1990)

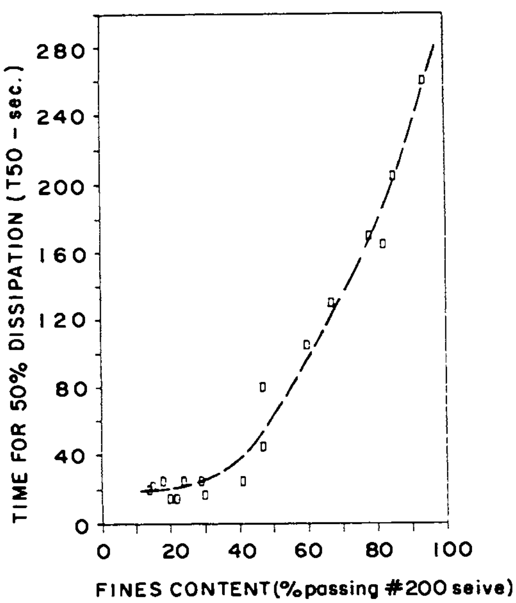

Fig. 11. Correlation Between Fines Content and T_{50}

greater than 40%. However, for T_{50} between 10 sec and 50 sec the cone penetration process is partially drained and there is a poor correlation between T_{50} and fines content.

LIQUEFACTION ASSESSMENT BY SHEAR WAVE VELOCITY

Shear wave velocity, V_s, is influenced by many of the variables that influence liquefaction potential, such as soil density, confinement, stress history and geologic age. Thus, V_s has promise as a field index in evaluating liquefaction susceptibility.

The major advantage of using shear wave velocity as an index of liquefaction resistance is that it can be measured in soils that are hard to sample such as silts and sands or hard to penetrate such as gravels.

The limiting shear wave velocities separating liquefied from non-liquefied sites for a given intensity and duration of shaking must be determined from field data. So far the data base is quite limited but it clearly shows that shear wave velocities may be a useful index of liquefaction potential. Data from sites in the Imperial Valley, California which liquefied during the 1979 Imperial Valley, 1981 Westmorland and 1987 Superstition Hills earthquakes suggest that the limiting shear wave velocity separating liquefiable from non-liquefiable sites has a lower bound in the region of 140 m/s for earthquakes of local magnitude M_L = 6.5 generating peak ground accelerations of about 0.17g (Holzer et al., 1988; Youd and Wieczorak, 1984).

Empirical methods have been developed to evaluate liquefaction resistance directly from shear wave velocity (Bierschwale and Stokoe, 1984; Stokoe and Nazarian, 1985; Tokimatsu et al., 1989). Over the past 15 years, significant advances have been made in measuring shear wave velocities in the field. Accurate and detailed profiles can be determined with conventional crosshole and downhole seismic methods (Stokoe and Hoar, 1978; Woods, 1978) or by spectral analysis of surface waves (Nazarian and Stokoe, 1986; Stokoe, 1990). A most significant advance in measuring in situ shear wave velocity in recent years has been the development of the seismic cone penetration test (Robertson et al 1986).

Robertson (1990) proposed a liquefaction assessment chart based on normalized shear wave velocity rather than $(N_1)_{60}$. Shear wave velocity is a function of void ratio and effective confining stress. Hence, for a sand of constant void ratio (constant density) the shear wave velocity will increase with increasing depth. Therefore, a correlation between V_s and the critical stress ratio (CSR) to cause liquefaction should be based on a shear wave velocity normalized with respect to effective overburden pressure, similar to the manner in which penetration resistance is normalized. Shear wave velocity being proportional to the square root of the shear modulus is a function of the effective overburden stress to the power 0.25, so

$$V_s = f[(\sigma_v')^{0.5}] \qquad (2)$$

Therefore, a normalized shear wave velocity, V_{s1}, can be established using the relationship

$$V_{s1} = V_s\ (P_a/\sigma'_{vo})^{0.25} \qquad (3)$$

where

P_a = reference stress, typically 100 kPa

σ'_{vo} = effective initial overburden stress in same units as P_a

The proposed correlation between normalized shear wave velocity and the critical cyclic stress ratio necessary to prevent liquefaction is shown in Fig. 12 (Finn et al., 1990a).

There is not enough data in Fig. 12 to define the role of earthquake magnitude. For similar τ/σ'_v ratios the role of magnitude, which is an index of duration, only becomes evident when case histories contain enough events in which liquefaction occurs towards the end of strong shaking.

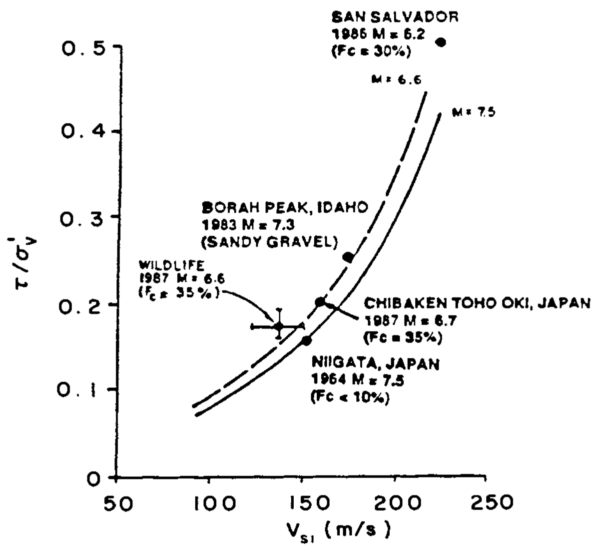

Fig. 12. Proposed Correlation Between Normalized Shear Wave Velocity and Cyclic Stress Ratio (CSR) to Cause Liquefaction

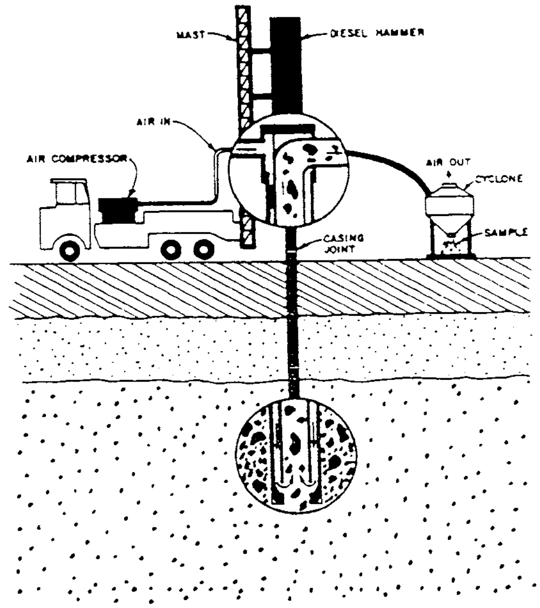

Fig. 13. Becker Penetration Test (Harder and Seed, 1986)

LIQUEFACTION POTENTIAL OF GRAVELS BY BECKER PENETRATION TEST (BPT)

The presence of large gravel and cobble particles preclude the use of the SPT in the evaluation of liquefaction potential. Misleading high SPT blowcounts would be recorded as the 2in sampler bounced off the large particles. A much larger and heavier penetration tool is required to penetrate gravels and provide a continuous record of penetration resistance. Harder and Seed (1986) developed the Becker Penetration Test (BPT) for this situation.

In the BPT a double-walled casing, generally 6.6 in OD in the U.S., is driven into the ground with a diesel pile hammer (Fig. 13). The casing can be driven with an open bit and reversed air circulation to obtain disturbed samples or with a plugged bit and driven as a solid penetrometer. The blows required for each foot of penetration are recorded and provide a record of penetration resistance for the entire profile. Two types of rigs are in use, the B-180 rig (also known as HAV-180) and the AP-1000 developed later in the 1970's. Both rigs use the same model of diesel hammer, an ICE Model 180. Harder and Seed (1986) showed that the older B-180 rig was more efficient with the result that the blow counts from AP-1000 rig were 50% higher than those from the B-180 rig. Harder and Seed (1986) developed the BPT procedures using the AP-1000 so blowcounts obtained by the B-180 rig must be adjusted to equivalent AP-1000 blowcounts unless correlations are developed specifically for the B-180 rig. Generally the blowcounts from the AP-1000 rig are 50% higher than from the B-180 rig.

Liquefaction potential is evaluated by first converting BPT blowcounts to equivalent SPT blowcounts using the correlation developed by Harder and Seed (1986) or a locally developed correlation and then following the procedures for using Seed's liquefaction assessment chart (Seed et al., 1985) described earlier. Before conversion to equivalent SPT blowcounts, the BPT blowcounts must be standardized for combustion conditions following the procedures described by Harder and Seed (1986). Perhaps consideration should be given to measuring directly the energy transferred to the casing and standardizing the blowcounts to an appropriate energy standard.

The BPT procedure has been used to evaluate the liquefaction potential of foundation gravels in the Mormon Island Dam near Sacramento, California. An excellent description of the use of the BPT on this project may be found in Hynes et al. (1988).

The BPT has been used in just a few projects so far and in North America only so there is yet very little field data by which to validate the procedure. Therefore a critical assessment is not possible at this time.

Some interesting questions surfaced during the past year in the assessment of BPT blowcounts from an ongoing project where the penetration exceeded 50 m. What is the effect of friction on the casing on blowcounts at deep penetrations? What is the best procedure for assessing the effect? There is clearly the possibility that high friction will result in unduly high blowcounts. This effect can be clarified by appropriate field testing.

Another question relates to the correlation between SPT and BPT. The correlation is established by conducting both tests in the same soil profile. Is this correlation still valid when BPT tests are run in deposits of gravel and cobbles? Even in relatively fine-grained soils, the correlation between SPT and CPT has been found to be dependent on mean grain size. A larger database on the liquefaction of gravels is needed to resolve this question. Of course, when such a database becomes available, a direct correlation between BPT and liquefaction potential can be established and the correlation between SPT and BPT ignored.

LIQUEFACTION POTENTIAL OF PLASTIC FINE-GRAINED SOILS

Liquefaction potential of plastic fine-grained soils are determined using the Chinese criteria developed by Wang (1979). These criteria are:

* per cent finer than 0.005 mm $\leq$ 20%
* liquid limit, LL $\leq$ 35%
* natural water content $\geq$ 0.9 LL
* liquidity index, $I_w \geq 0.75$

Soils which satisfy all four criteria are judged vulnerable to liquefaction or significant strength loss. In addition, any fine grained soils for which the standard penetration resistance N $\leq$ 4 are considered to be vulnerable to liquefaction or to significant strength loss under cyclic loading whether or not they satisfy the Chinese criteria (Woodward Clyde Consultants, 1990). The Chinese criteria are usually applied strictly with no account taken of uncertainties in the measurements of the parameters in the criteria.

In connection with investigations of the liquefaction potential of Sardis Dam in Mississippi (Finn et al., 1990b), Woodward Clyde Consultants (1989) suggested that allowances should be made for uncertainties in the measured values of the parameters in the criteria. Following a study of Wang's database, they recommended ignoring the liquidity index and making the following changes in the measured soil properties before applying the criteria:

* decrease per cent fines by 15%
* decrease LL by 5%
* increase water content by 3%.

These changes increased significantly the extent of the soils vulnerable to liquefaction and strength loss so that almost the entire length of dam required remediation. Therefore, engineers of the Vicksburg District of the U.S. Army Corps of Engineers, who are responsible for the safety assessment and remediation of Sardis Dam, reviewed reports on the scatter in measured index properties in U.S. Corps of Engineers' laboratories over the last 30 years to determine the likely range of variations in test data. In addition they conducted tests on samples of standard soils of low to medium plasticity in their own laboratory to establish the scatter in their own data. These standard soils are used to check comparability of testing procedures and results between different Corps of Engineers' laboratories and private laboratories. As a result of these studies the following changes in measured properties were adopted before applying the Chinese criteria (again ignoring the liquidity index):

* decrease the fines content by 5%
* decrease the liquid limit by 2%
* increase the water content by 2%.

This change reduced the length requiring remediation to about 1220 m (4000 ft).

The impact of the Chinese criteria on the extent of remediation necessary for stability appeared to be so critical that an investigation of Chinese procedures was undertaken by Koester (1990). The Chinese determine the liquid limit using a fall cone rather than the Casagrande device generally used in North America. Using a standard Chinese fall cone and following Chinese standard SD 128-007-84, Koester (1990) showed that the fall cone gives a liquid limit about 3% to 4% greater than the Casagrande device. The Koester study is not yet complete and findings relative to the liquid limit should be viewed as tentative.

On the basis of all the above studies the following changes in measured index properties were finally adopted to account for uncertainty before application of the Chinese criteria:

* decrease the fines content by 5%.
* increase the liquid limit by 1%
* increase the water content by 2%

These changes reduced the length requiring remediation to about 926 m (3000 ft).

Clearly the Chinese criteria for evaluating the potential for liquefaction or significant strength loss in clayey soils, based on liquid limit, water content and per cent fines $\leq$ 0.005 mm, can have a major impact on the extent of remedial measures necessary to achieve stability in earth structures with potentially liquefiable fine grained materials.

Before applying the Chinese criteria the uncertainties in the measured soil properties should be taken into account in a reasonably conservative manner. This may be done by adjusting the measured water content, liquid limit, and the fines content before applying the criteria. The amount of these adjustments should be based preferably on the estimated variability in data appropriate for the laboratory conducting the tests. In the absence of such specific information, the adjustments noted above of increasing the liquid limit by 1%, the water content by 2%, and decreasing the fines content by 5% may be considered. These adjustments reflect conservative estimates of the variability to be expected from very experienced personnel operating under high standards of quality control.

DETERMINATION OF STEADY STATE STRENGTH

S_{us} from Laboratory Tests

The steady state strength is the major factor controlling post-liquefaction behaviour. Liquefaction is synonymous with strain softening in undrained shear as illustrated by curve 1 in Fig. 14. When soil is

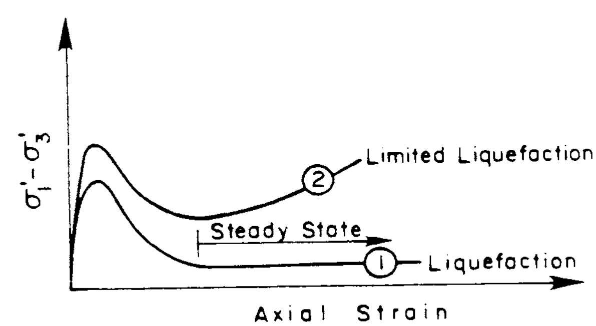

Fig. 14. Types of Contractive Deformation

strained beyond the point of peak strength, the undrained strength drops to a value that is maintained constant over a large range in strain. This is called the undrained steady state or residual strength. If the strength increases after passing through a minimum value, the phenomenon is called limited liquefaction and is illustrated by curve 2 in Fig. 14. Limited liquefaction may also result in significant deformations because of the strains necessary to develop the strength to restore stability.

The steady state strength of liquefied soils, S_{us}, cannot be determined by undrained shear tests on "undisturbed" samples from the field. Such contractive soils are very difficult to sample. They are likely to densify during sampling, transportation and the process of setting up the tests. Therefore the tests cannot be conducted at the field void ratio. A procedure for dealing with this problem has been reported by Poulos et al. (1985) which is shown in Fig. 15.

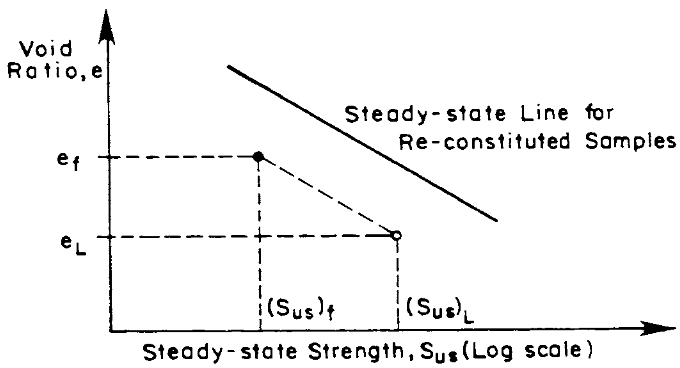

Fig. 15. Procedure for Estimating Steady State Strength (Poulos et al., 1985)

The steady state line in e-S_{us} space is obtained by tests on reconstituted samples at different void ratios. Then S_{us} is measured in tests on good quality undisturbed samples from the field. A representative value of this laboratory steady state strength, $(S_{us})_L$ is plotted in Fig. 16 at the void ratio at failure in

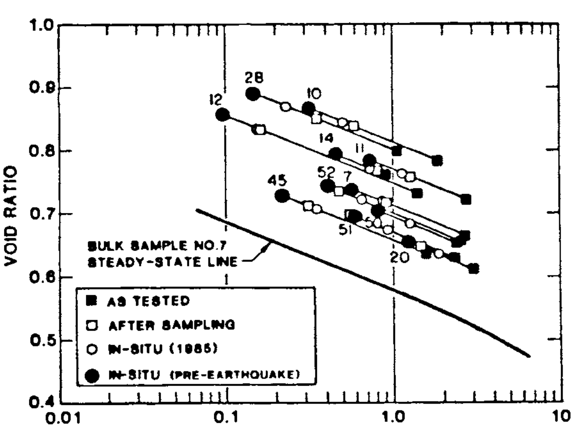

Fig. 16. Variation in Steady State Strengths of Field Samples (Seed et al., 1988)

the laboratory, e_L. The steady state line for field conditions is assumed to be parallel to the line for reconstituted samples. Therefore a line is drawn through the point $((S_{us})_L, e_L)$ parallel to the line for reconstituted samples and the strength corresponding to the field void ratio, e_f, is taken as the steady state strength in the field, $(S_{us})_f$. The field void ratio is determined by applying a complex series of corrections to the void ratio of the consolidated test specimen.

The differences between the steady state lines for good quality "undisturbed" samples can be very wide as shown in Fig. 16 and therefore the selection of a representative steady state strength poses very significant problems for a designer. It should also be noted that in this case the steady state line for the reconstituted samples are much lower than those for the "undisturbed" samples.

The differences in steady state strength of the undisturbed samples may be attributed substantially to differences in gradations between the samples, for it has been well established that the steady state strength can vary widely with variations in gradation. However, there is an additional difference between the samples constituted from Bulk Sample 7 and the undisturbed samples. The test specimens fabricated from Bulk Sample 7 were prepared by moist tamping. This compaction procedure gives a relatively isotropic structure, whereas the hydraulically deposited field samples seem to have a more anisotropic structure. Kuerbis and Vaid (1989) concluded that moist tamping imparts a fabric that promotes contractive behaviour and at void ratios that would not otherwise be contractive. For this reason moist tamping does not seem an appropriate technique for sample preparation. The slurry preparation technique of Kuerbis and Vaid (1989) seems to model field conditions much better for soils with fines.

Byrne (1990) has presented data from practice that tends to confirm dramatically the effects of gradation. He established the steady state lines for a silty sand, and a sandy silt occurring in a layered deposit of tailings and for a mixture of the two materials. At constant water content the steady state strength of the mixture was significantly less than that of either the sandy silt or the silty sand. At a water content of $w = 24\%$, the steady state strength of the mixture was approximately zero while the strengths of the silty sand and the sandy silt were 100 kPa and 300 kPa respectively.

The steady state strength is considered to be independent of the stress path (Castro et al., 1989). Vaid et al. (1989) conducted an extensive test program on Ottawa sand and a tailings sand to investigate the effects of stress path on steady state strength using extension and compression tests. They found that the steady state strength was greatest in compression. In the case of Ottawa sand, the ratio of compressive strength to the extension strength was 10 to 1; for the tailings sand the ratio is about 6:1. Furthermore they found that the range in void ratio exhibiting contractive behaviour in extension is much larger than in compression. These tests were conducted on water pluviated sands to simulate the depositional process in nature. Such sands are inherently anisotropic and their response to loading depends on the orientation, β, of the major principal stress in relation to the plane of deposition. The data from Vaid et al. (1989) strongly suggest that the steady state or residual strength is a function of stress path as specified by the angle β.

The dependence of S_{us} on β has important practical implications. The angle β varies along the curved failure surface of an embankment from $\beta = 0$ (compression loading) near the crest to $\beta = 90°$ (extension loading) at the toe. Therefore the steady state strength as measured by the Poulos procedure (Poulos et al., 1985) would appear to be applicable only near the upper part of the failure surface. The strength should decrease and reach its lowest value in extension near the toe. Thus the average steady state strength may be much less than the value measured in laboratory compression tests. Seed et al. (1988) have shown from back analysis of the San Fernando dam that the average steady state strength in situ was substantially less than the average strength measured by the Poulos procedure. The effect may be even more dramatic in the case of the San Fernando dam failure because the potential surface of sliding in the deformed state is largely horizontal (Fig. 17).

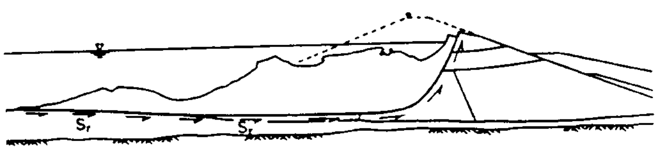

Fig. 17. Post-Liquefaction Deformed Shape of Lower San Fernando Dam (Seed et al., 1988)

Discrepancies between steady state strength in compression measured on undisturbed samples and the average steady state strength computed by back analysis of the San Fernando slide are credited to densification of the samples of loose materials during sampling and handling. The quantitative effect of stress path on steady state strength suggested by the work of Vaid et al. (1989) may account for a substantial part of the difference noted in the San Fernando studies. This effect is also crucial to a reliable stability analysis.

There are clearly sharp differences between recent research findings and current practice in the determination of steady state strength from laboratory tests. The key assumption underlying current practice that the steady state strength is a function of void ratio only needs further investigation. More studies on the effects of stress path and fabric (method of sample preparation) are needed to establish a generally acceptable position on this very important problem.

S_{us} from Case Histories

Seed (1987) developed an alternative approach to determining steady state strength. He analyzed the stability of a number of field cases in which large deformations occurred after liquefaction and developed a correlation between in situ steady state strength and representative $(N_1)_{60}$ values. This correlation was updated in 1988 by refining some of the previous analyses and incorporating new data from embankment failures during the Chilean earthquake of 1985 (De Alba et al., 1987). The latest version of the correlation is shown in Fig. 18.

These strengths were deduced from stability analyses of potential sliding along assumed or actual slip surfaces. Therefore energy dissipation was assumed to occur only along the sliding surface due to the relative motion between the two rigid blocks.

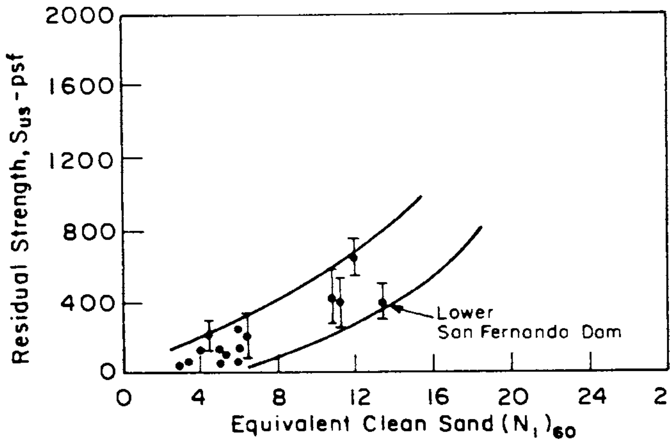

Fig. 18. Relationship Between Residual Strength and $(N_1)_{60}$ (Seed et al., 1988)

However when a substantial zone of liquefaction exists, as in the case of the San Fernando Dam, energy can be dissipated by shearing deformations within the liquefied zone. Neglecting this may lead to an overestimation of the residual strength. An analogous situation exists in the radial shear zone under a footing undergoing failure in undrained shear in cohesive soil under a failure pressure q_f. If the undrained strength were deduced by considering only sliding along the boundaries of the failure zones and neglecting the energy dissipated in the zone of radial shear, a much higher value for the cohesion, C_u would be obtained than from the exact Prandtl solution $C_u = q_f/(2+\pi)$ which includes energy dissipation in the zone of radial shear. Though this effect may be important if the zone of liquefaction is substantial, there seems to be no way at present by which it may be estimated.

The role of diffusion of porewater under seismically induced gradients is usually not included in the analysis of case histories. Delayed failures such as that of the San Fernando Dam (Seed et al., 1988) and the Mochikoshi Tailings Dam (Ishihara et al., 1978; Finn, 1982) have been attributed to the migration of porewater towards dilating zones of low porewater pressures with a consequent reduction in available strength.

If the liquefied zone is surrounded by a relatively impermeable zone, then free porewater may accumulate under the impermeable layer as the liquefied materials consolidate. This phenomenon was evident in the sealed layers in the shake table tests of Finn et al. (1971) and especially in the special centrifuge tests of Arulanandan (1988). If this accumulation occurred in the field the consequences for stability would be disastrous. However, in earth structures, cracking of the stiffer material above the liquefied zone would tend to allow some drainage of the free water.

The above considerations create a substantial measure of uncertainty in the analysis of case histories and may be contributing to the scatter in the database which relates penetration resistance to steady state strength.

An excellent discussion of the difficulties associated with the analysis of case histories of flow deformation to determine steady state strength may be found in Seed et al. (1988).

Marcuson et al. (1990) reviewed the problems associated with the determination of steady state strength with the following conclusion, "the empirical correlation relating penetration resistance to field performance is adequate, and even may be preferable to the testing of undisturbed samples".

However, there are two challenges for the designer in using this correlation. The first is related to the range in strength at a given penetration resistance. At the low penetration resistances associated with very contractive materials the range in strength is many times the minimum value. This makes it very difficult to decide on an appropriate value for residual strength at low penetration resistances. The second challenge relates to the selection of a representative $(N_1)_{60}$. The dispersion of $(N_1)_{60}$ values can be very wide as illustrated by the data in Fig. 19 from the San Fernando Dam study (Seed et al., 1988). This dispersion can be particularly troubling when analyzing case histories to determine steady state strengths.

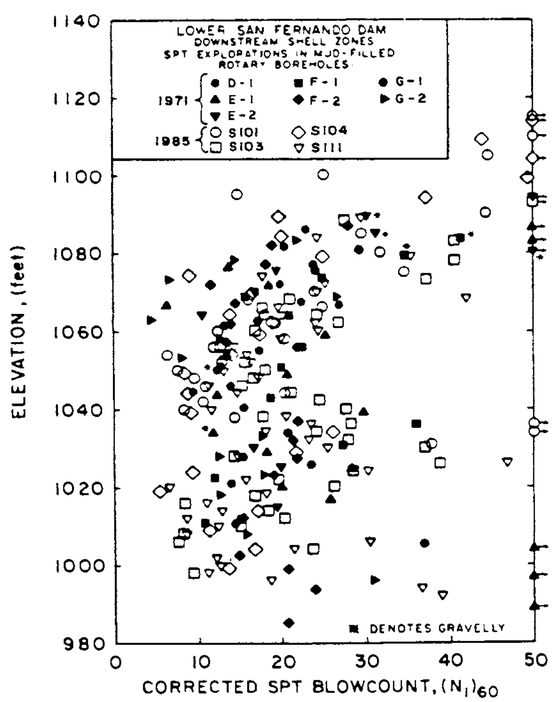

Fig. 19. Variation in $(N_1)_{60}$ Values in San Fernando Dam, 1971 and 1985 Investigations (Seed et al., 1988)

A final important question is whether the design earthquake can trigger liquefaction. Castro et al. (1989) use laboratory tests to establish the strain necessary to trigger liquefaction and then conduct a Newmark (1965) type of analysis to determine whether this strain will be achieved. In practice triggering is often determined by the Seed liquefaction assessment charts in Fig. 1 or Fig. 8. If liquefaction in the sense of these charts is predicted for contractive sands then it is assumed that the residual strength is triggered. However for a finite element analysis of post-liquefaction behaviour a triggering criterion is required for each finite element. Possible criteria are the peak strain criterion referred to above and the stress ratio criterion proposed by Vaid and Chern (1985).

ANALYSIS OF POST-LIQUEFACTION BEHAVIOUR

An important consideration in evaluating the seismic stability of an earth structure such as an embankment dam with potentially liquefiable materials is whether a flow failure will take place if liquefaction occurs. This will depend on the relationship between the average driving shear stresses in the dam along any potential failure surface and the corresponding steady state or residual strengths of the liquefied materials (Castro et al., 1985). If the driving shear stresses due to gravity on a potential slip surface through liquefied materials in an embankment are greater than the undrained residual strength, deformations will occur until the driving stresses are reduced to values compatible with static equilibrium. The more the driving stresses exceed the steady state strength, the greater the deformations to achieve equilibrium. Post-liquefaction equilibrium is evaluated using conventional static stability analysis to determine the factor of safety. Remedial measures are usually required if the factor of safety drops below some acceptable value such as 1.1.

The main steps in the current procedure for evaluating if liquefaction will occur and the seismic behaviour of soil structures containing liquefied zones are:

* Establish the maximum credible earthquake for design by appropriate geological and seismological investigations.

* Determine which soils will liquefy on the basis of in situ penetration test data and dynamic analysis following procedures set out by Seed (1983), Seed et al. (1985).

* Determine the residual or steady state strengths of the liquefied soils following procedures described by Castro et al. (1989) or by Seed (1987), revised by Seed et al. (1988) and Seed and Harder (1990).

* Establish appropriate strengths for all other materials.

* Conduct a conventional slope stability analysis incorporating the post-earthquake strengths of all soils to determine the factor of safety of the soil structure in its original configuration. Residual strengths are used for liquefied soils. For other soils the pre-earthquake strengths are reduced to account for the effects of seismically induced porewater pressures in sands and of cyclic loading in the case of clays.

* If the factor of safety is less than an acceptable value plan remedial measures.

* Estimate the deformations of the structure to ensure that they are within tolerable limits even if the post-liquefaction factor of safety is acceptable. Deformations are usually estimated using the rigid block analysis of

Newmark (1965) or the semi-empirical method developed by Seed et al. (1973) based on equivalent linear analysis.

* Depending on the factor of safety of the remediated structure and the type of remediation, it may be advisable to check the deformations of the remediated structure under design earthquake loading.

An alternative approach, more in keeping with the concept of designing dams for acceptable deformations proposed by Newmark (1965), is to evaluate the extent of necessary remedial measures on the basis of a tolerable amount of deformation for the low probability event specified by the design earthquake. The potential post-liquefaction deformations before and after remediation may be estimated using the computer program, TARA-3FL (Finn and Yogendrakumar, 1989), which is a specialized derivative of the general program TARA-3 by Finn et al. (1986). This deformation approach was adopted for the Sardis Dam studies as a complement to the factor of safety approach.

Structure of the Program TARA-3FL

The basic theory of the finite element program TARA-3 has been reported by Finn (1985,1990) and Finn et al. (1990b,c). Only procedures specific to TARA-3FL will be described here. The first requirement is a triggering criterion to switch the strength of any liquefiable element in the soil structure to the steady state strength at the proper time during the dynamic analysis. Two criteria are available, the peak strain criterion of Castro et al. (1989) and the stress ratio criterion of Vaid and Chern (1985). A useful approximation is to assume that the residual strength will be triggered in all elements that will liquefy according to the criteria developed by Seed (1983) and Seed et al. (1985).

In a particular element in the soil structure, the shear stress-shear strain state which reflects pre-earthquake conditions is specified by a point P_0 on the stress-strain curve as shown in Fig. 20. When liquefaction is triggered, the strength will drop to the steady-state value. The post-liquefaction stress-strain curve cannot now sustain the pre-earthquake stress-strain condition and the unbalanced shear stresses are redistributed throughout the dam. In the liquefied elements, the stresses are adjusted according to the following equation,

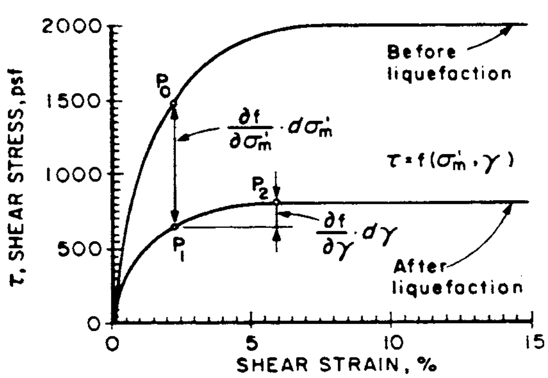

Fig. 20. Adjusting Stress-Strain State to Post-Liquefaction Conditions

$$\partial \tau = \frac{\partial f}{\partial \sigma'_m} d\sigma'_m + \frac{\partial f}{\partial \gamma} d\gamma \tag{2}$$

where $\tau = f(\sigma'_m, \gamma)$. This process leads to progressive deformation of the dam until equilibrium is reached at the state represented by P_2.

Since the deformations may become large, it is necessary to update progressively the finite element mesh. Each calculation of incremental deformation is based on the current shape of the dam, not the initial shape as in conventional finite element analysis. An independent assessment of the equilibrium of the final position should be conducted using a conventional static stability analysis. The factor of safety determined in this way should be unity or greater depending on whether the deformations occurred relatively slowly after the earthquake or during it when seismic inertia forces are acting.

CASE HISTORIES

Deformation Analysis of Sardis Dam, Mississippi

Preliminary deformation studies of Sardis Dam have been conducted using TARA-3FL for various assumptions about the magnitudes and distributions of the residual strengths in the liquefied zones. Evaluations of dam performance during seismic shaking and potential remedial measures are still underway, so final results cannot be presented here. Instead some results are presented to show the type of information provided to the design engineer by deformation analyses.

The general configuration of the Sardis Dam is shown in Fig. 21. During the design earthquake, liquefaction is predicted to occur in the core and in a thin seam of clayey silt in the top stratum clay in the foundation.

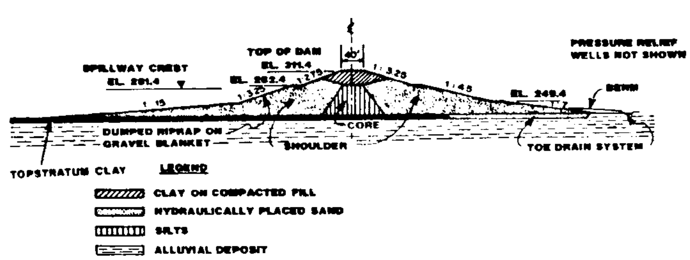

Fig. 21. Typical Section of Sardis Dam

The thin layer may be seen clearly in Fig. 22. The liquefaction potential was evaluated using Seed's liquefaction assessment chart (Seed et al., 1985) and the Chinese criteria for soils with plastic fines by Wang (1979).

The residual strength in the core was assumed to be 5 kPa (100 psf) based on Seed's correlation between corrected standard penetration resistance $(N_1)_{60}$ and residual strength (Seed, 1987; De Alba et al., 1987). From a variety of studies, the residual strength in the thin layer in the foundation was assumed to be 0.075 σ'_{vo} (Woodward Clyde Consultants, 1989). The original strength of the thin layer was taken as 100 kPa (2000 psf).

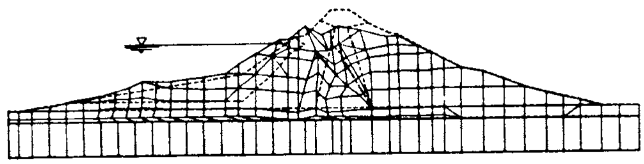

Fig. 22. Initial and Post-Liquefaction Configurations of Sardis Dam

The large differences between the initial and post-liquefaction strengths in Sardis Dam resulted in major load shedding from liquefied elements. This put heavy demands on the ability of the program to track accurately what was happening and on the stability of the algorithms. Therefore it was imperative to have an independent check that the computed final deformed positions were indeed equilibrium positions. The most direct check is to run a conventional stability analysis on the deformed position. If the major deformations occur during the earthquake, the resulting factor of safety should be greater than unity because some of the deformation field is driven by the inertia forces. If the major deformations occur relatively slowly after the earthquake, the factor of safety should be close to unity. A number of delayed post-earthquake failures has occurred, the most notable being the failure of the Lower San Fernando Dam during the 1971 San Fernando earthquake (Seed, 1979).

In one analysis of Sardis Dam, the residual strength values specified above were used, but assuming a minimum value of 17.5 kPa (350 psf) in the thin liquefied layer in the foundation. The initial and final deformed shapes of the dam for this case are shown in Fig. 22. Very substantial vertical and horizontal deformations may be noted, together with intense shear straining in the weak thin layer. The static stability of the deformed shape was analyzed using the program UTEXAS2 (USACE, 1989). This program uses Spencer's method (1973) which satisfies both moment and force equilibrium. The factor of safety was found to be close to 1.0. It is also interesting to note that the critical slip surface for UTEXAS2 analysis exited the slope near the location suggested by the finite element analysis using TARA-3FL.

The reliability of TARA-3FL in predicting stable deformed shapes was tested by parametric studies with various assumptions about the steady state strength in the thin liquefiable layer. The factors of safety of the undeformed and deformed dam cross-sections were determined using UTEXAS2. The factors of safety for the undeformed dam are given by the solid sloping line in Fig. 23. The points give the factors of safety of the post-liquefaction deformed sections. The steady state strengths in Fig. 23 represent either a constant value for the layer or an imposed minimum value when $S_{us} = 0.075\ \sigma'_{vo}$. In the clearly unstable region defined by a factor of safety less than one for the undeformed section, the computed factors of safety for the deformed sections were in the range of 1 ± 0.05. This is the theoretical error band associated with UTEXAS2.

Many results of this type, for different assumptions about the residual strengths, suggest that the TARA-3FL analysis does indeed achieve equilibrium positions even for large drops in strength due to liquefaction.

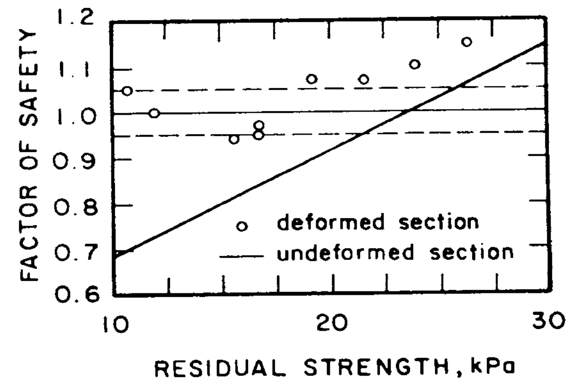

Fig. 23 Variations in the Factor of Safety with Residual Strength

Studies were made of the sensitivity of displacements to various levels of residual strength in the thin layer. The variations in the vertical displacements at the upstream edge of the crest (curve 1) and in the horizontal deformations at the midpoint of the upstream slope (curve 2) are shown for various levels of constant residual strength in the thin liquefied layer in the foundation in Fig. 24. The increase in displacement is gradual with decrease in residual strength until the strength drops to about 20 kPa (400 psf) when the displacements begin to increase very rapidly.

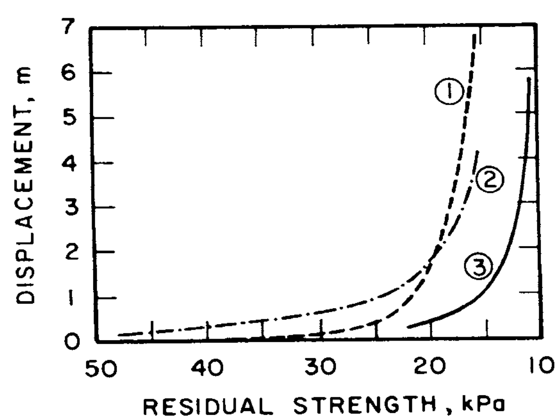

Fig. 24. Variation in Displacement with Residual Strength

The variation in vertical displacements (curve 3) is also shown, Fig. 24, for residual strengths $S_r = 0.075\ \sigma'_{vo}$. For variable minimum residual strengths, the displacements increase rapidly when the minimum strength is about 15 kPa (300 psf).

It is also possible to determine the loss in freeboard associated with various factors of safety based on the original configuration of the dam for various residual strengths. The variation of vertical crest displacement with factors of safety of the undeformed dam are shown for various values of residual strength in Fig. 25. For the first time a designer has available the deformation fields associated with different factors of safety for a particular dam. This information is helpful in deciding on an appropriate factor of safety.

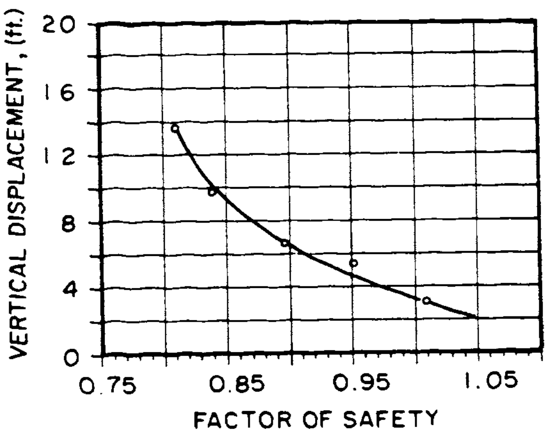

Fig. 25. Variation of Vertical Displacement with Factor of Safety

ROLE OF ANALYSIS IN PLANNING REMEDIAL MEASURES

Remedial measures for Sardis Dam will be limited to an upstream plug crossing the weak clay layer in the foundation clay stratum as shown in Fig. 26. The

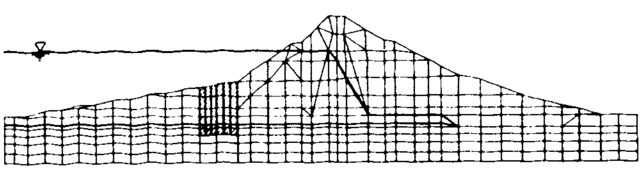

Fig. 26. Sardis Dam with Remediation Plug

closest proximity of the plug to the core of the dam will be adjacent to the slope break on the upstream slope. This location is controlled by the conservation level of the pool during construction. Note that a finer finite element mesh (Fig. 26) is being used for evaluating the effectiveness of remedial measures than was used in the original deformation studies (Fig. 22) in order to provide a more accurate description of stresses and strains within the remediated section.

During shaking by the design earthquake, the saturated portion of the core and the weak foundation clay outside the remediated plug are still expected to liquefy. Therefore the plug must fulfill two functions: it must have sufficient strength to prevent shearing along the level of the weak clay layer and also have sufficient stiffness to prevent significant loss of freeboard. If the plug is not stiff enough, bending deformations, with or without cracking of the plug, may allow significant crest settlements. Therefore the aim of TARA-3FL analyses of remediated sections is to establish the strength and stiffness combinations that will limit loss of freeboard as economically as possible.

For example, if the remedial plug in Fig. 26, has a shear strength of 100 kPa (1 tsf) and a shear modulus of 3 x 10^4 kPa (6 x 10^5 psf) there is an anticipated loss of freeboard of about 0.5 m (1.5 ft). The deformed shape is shown in Fig. 27. Note the significant distortion in the liquefied core and in the weak foundation layer.

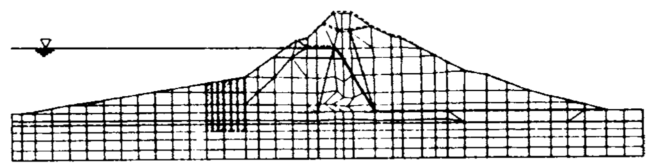

Fig. 27. Post-Liquefaction Deformations in Remediated Section of Sardis Dam

The thickness of the weak clay layer is variable ranging from about 1.2 m (4 ft) up to 6 m (20 ft). Clearly for the greater thickness of weak foundation layer, either the stiffness or width of the remedial plug must be increased. TARA-3FL analyses are now being conducted to establish the properties of the remedial plug to limit loss of freeboard to a tolerable value as a function of the thickness of the weak foundation layer. The next step in the remediation process is to select a method of achieving these properties in the field. A detailed study of the advantages and disadvantages of various remedial measures for Sardis Dam has been conducted which took into consideration not only engineering factors but factors such as cost, reliability, availability of experienced contractors and the conservation level of the pool. The final selection of a method of remediation will be made when data from construction test sections become available.

Ground Deformations in Niigata City

Very large ground deformations occurred in liquefied soils in Niigata City, Japan, during the 1964 earthquake. Hamada et al. (1988) conducted detailed studies of these deformations in three zones of the city, shown as A, B, and C, in Fig. 28. The vector displacement field for zone B, shown in Fig. 29, was developed on the basis of aerial photographs taken before and after the earthquake.

A cross-section of the ground between Ogata School and the Tsusen River identified as L-L' in Fig. 26 is shown in Fig. 30, together with the distribution of displacements along the section. Peak displacements are about 7 m. Ogata school is a stationary point with the displacements leading from it in opposite directions. The Tsusen River provided a free surface which facilitated displacements of gently sloping

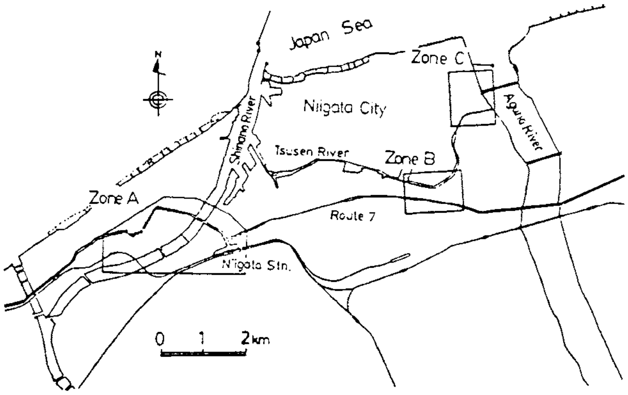

Fig. 28. Zones where Deformations were Studied in Niigata City (Hamada et al., 1988)

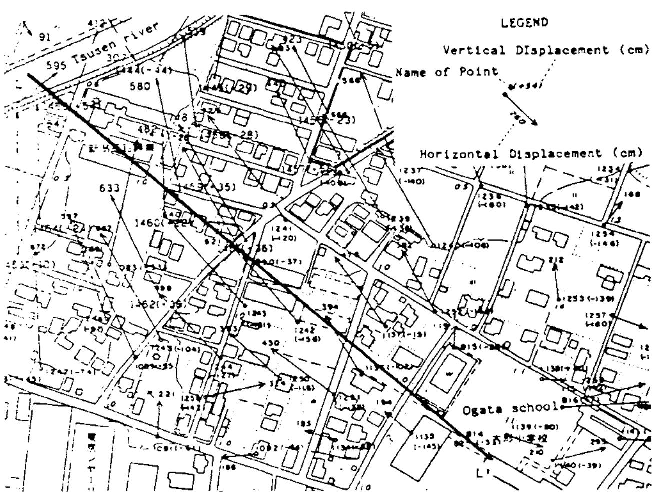

Fig. 29. Vector Displacement Field in Zone B of Niigata City (Hamada et al., 1988)

ground. There were reports that there was considerable inflow into the river which quickly eroded away after the earthquake.

A deformation analysis was conducted on section L-L' using a simplified model of the topography and of the geometry of the liquefied layer.

The residual strengths were selected on the basis of the N-values shown in Fig. 30, using the correlation by Seed et al. (1988). As pointed out earlier, at very low penetration resistances, there is great uncertainty in selecting an appropriate residual strength. In this case, with very low driving shear stresses and large measured deformations, it was clear that the minimum values from the correlation would be appropriate.

The computed deformations are shown by the solid curve in Fig. 30. Predictions are good except at the

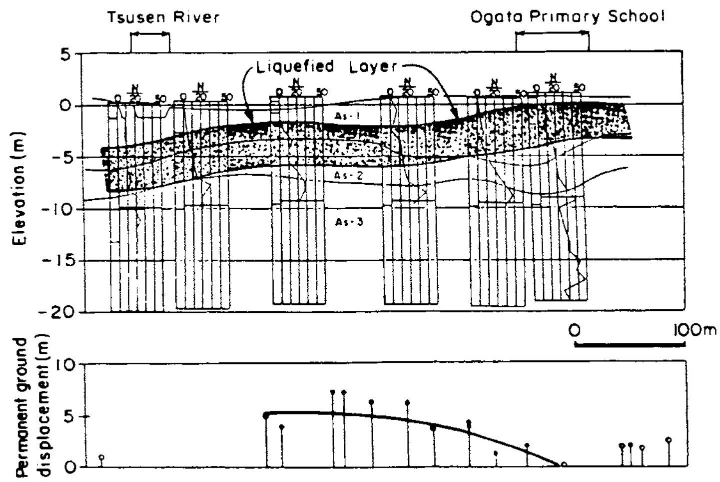

Fig. 30. Section L-L' Showing Topography Liquefied Layer and Measured and Computed Displacements

location of maximum displacements. Part of the reason for low computed values in this area may be that an average slope was assigned to the liquefied layer rather than the actual slightly undulating form. Therefore the slope leading into the area of peak displacements in the calculations was somewhat flatter than the original slope.

These results are very tentative because of the uncertainties in the residual strength and the simplified geometry of the section used in the analysis. Nevertheless over a section approximately 0.5 km in length, the pattern of computed displacements is representative of the displacements that occurred during the earthquake.

SUMMARY OF DEVELOPMENTS

The standard penetration test remains the primary tool for evaluating liquefaction potential. The procedures for running the test and processing the results to obtain representative values of $(N_1)_{60}$ are now well defined. The adoption of an energy standard of 60% of the energy of the hammer in free fall has done much to make the test a standard test.

There is an extensive database relating $(N_1)_{60}$ to liquefaction resistance. However the database relates primarily to shallow level ground sites. In addition, the surface manifestations of liquefaction in the database such as sand boils are mainly indicative of the development of high porewater pressure. There is little direct evidence whether the sands were contractive or not.

In many practical applications the method is extended to depths many times that reflected in the database and to conditions with significant initial static shear using correction factor K_σ and K_α. These extensions are logical and based on laboratory test data but there is little direct evidence from the field to validate their application to the more extreme conditions. Also the pronounced element of judgement involved in evaluating case histories makes it difficult to assess the role of initial static shear in the field.

The cone penetration test is emerging as an attractive alternative to the SPT because it is much easier to conduct in a standard manner. Field tests indicate a high degree of repeatability. Furthermore it gives a continuous penetration record and better definition of thin liquefiable layers. The current drawback to the test is the lack of an adequate database to relate the CPT cone bearing to liquefaction resistance directly. At present liquefaction assessment charts base on CPT data rely on the conversion of the SPT database to a CPT database by correlations between SPT N-values and Q_c from CPT tests. This conversion introduces additional uncertainty into the liquefaction assessment charts. Although the cone is very useful in identifying soils in a general way during penetration, samples must be recovered independently for more precise identification since the penetration resistance is an index of liquefaction potential only for non-plastic soils.

The Becker Penetration Test (BPT) involving driving heavy casing with a diesel hammer, has been introduced to determine the penetration resistance deposits with particles of gravel or cobble sizes. The liquefaction resistance is assessed using the SPT database. The correlation between SPT and BPT resistances is established by running both tests in sands. It is assumed that this correlation is valid when the BPT is run in deposits where SPT cannot be conducted because of particle size. This is a major assumption which cannot be verified until a sufficient database has been developed to allow a direct correlation between BPT resistances and liquefaction resistance.

An unresolved issue with the BPT for large depths is the effect of casing friction on penetration resistance and on the correlation between SPT and BPT resistances. This has become a troublesome point in some recent applications. Procedures need to be agreed for estimating the friction.

Shear wave velocity has been advanced as an alternative to penetration resistance as an index of liquefaction resistance. It appears to be an attractive option for use in hard to sample or hard to penetrate soils. The spectral analysis of surface waves is a technique that makes the method applicable to gravels and cobbles in which conventional methods of shear wave velocity measurement would be impossible or very difficult. The development of the seismic cone has made the measurement of shear wave velocity in sands and silts a very simple procedure which can be conducted in conjunction with normal CPT investigations. The main drawback to the shear wave velocity approach is the lack of an adequate database to provide direct correlation between shear wave velocity and liquefaction resistance.

The liquefaction potential of weak silty clays or clayey silts of low plasticity is assessed using the Chinese criteria. Experience with these criteria in practice indicates that the extent of the zone of liquefaction is sensitive to variations in the parameters used in the criteria. In the application of these methods in practice, therefore, uncertainties in the measurements of liquid limit, fines content and water content should be taken into account. As a preliminary guide the measured fines content may be reduced by 5%, the water content increased by 1% and the liquid limit decreased by 2% to account for uncertainty.

The major factor controlling post-liquefaction behaviour is the residual or steady state strength. Two methods have evolved for determining it; laboratory tests on high quality samples and a correlation between $(N_1)_{60}$ and steady state strength. Laboratory procedures involve complex corrections to determine the field void ratio. Both procedures involve major uncertainties. The determination of steady state strength from $(N_1)_{60}$ data appears to be the preferred approach at the present state of knowledge.

Deformation analysis of earth structures with liquefied zones is a very useful complement to conventional slope stability analysis for studying the effects of liquefaction and designing remedial measures. It provides the designer with additional useful information which assists him in exercising his judgement effectively. It is particularly useful when only partial remedial measures are employed and liquefaction is still permitted to occur in other sections of the structure. In these cases not only the global stability guaranteed by conventional stability analysis is important but also the localized displacements that occur in the liquefied sections.

Analysis of post-liquefaction deformations must cope with large changes in strength and stiffness of the soils when liquefaction is triggered. These changes make major demands on the accuracy and stability of the method of analysis. Therefore it is imperative that the stability of any computed deformed section be checked by a conventional stability analysis.

Deformation analyses of Sardis Dam were computed by the program TARA-3FL for a wide range in post-liquefaction strength and stiffness of the weak clays in the foundations. The factors of safety of the deformed sections computed by an accurate conventional stability analysis are close to unity. They lie within the established error bounds for the stability analysis. This confirms that TARA-3FL does indeed compute deformed equilibrium configurations.

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